Sliding failure analysis of a gabion retaining wall at km 31+800 of Lubuk Selasih – Padang city border highway, Indonesia

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Abstract, In August 2010, there was a landslide on the down-slope of national road section at Km 31+800 Lubuk Selasih – Padang City Border. In order to prevent further damage, it was necessary to make an immediate repair by constructing a gabion retaining wall. Since this repair was so urgent, physical and mechanical soil parameters for the stability analysis were determined from literature data. The stability analysis considered dangers of overturning, sliding, and soil bearing capacity. For the sliding stability analysis, the value for friction considered only the interaction between the soil and the base of the retaining wall, with the assumption that the contact area was equal to the total area of the entire base of the retaining wall. After the construction was completed, sliding failure occured due to pressure from the backfill embankment. This research performs a reanalysis of the retaining wall stability using soil and gabion parameters determined from field investigation and laboratory testing. In this reanalysis the friction contact area was assumed to be between the soil and the wire mesh of retaining wall. With these parameters and assumption, the main cause of sliding failure became clear, indicating that this approach increased the accuracy of stability analysis for gabion retaining walls.

1.Introduction

Instability of earth retaining walls is a serious problem in geotechnical engineering, with considerable potential damages on failure. Therefore, it is vital that the causes of instability be understood.

The failure of retaining walls has been extensively researched. A study in Vietnam of concrete retaining wall stability looked at sliding, rotation, and vertical deformation caused by fluctuations in groundwater [1]. Instability caused by the construction failing to conform to the design based on prescriptive guidelines in a concrete retaining wall in India resulted in sliding, rotation, and vertical deformation [2]. An investigation was performed into a 6m high gabion retaining wall in Canada that experienced vertical deformation and sliding three years after the construction was completed, and found the instability was caused by the lack of bearing capacity analysis[3]. Stability analysis of a collapsed cantilever retaining wall in Lembah Anai, Indonesia found the lack of consideration of the dimensions in the design was the main factor contributing to the instability[4]. Excessive deformation of a gabion retaining wall in Johannesburg, South Africa was investigated and the lack of bearing capacity analysis was found to the the main factor along with underestimates of lateral earth pressure[5]. An investigation into a collapsed gabion retaining wall in Byreburnfoot,Scotland, and found the joints between basket couldn"t resist the lateral earth pressure from granular backfill [6].

This paper is a post failure analysis of a gabion retaining wall on a section of the Lubuk Selasih – Padang City border national highway at km 31+800 from Padang. In this case, the retaining wall experienced an excessive sliding failure just after the construction was completed. It showed no other sign of failure such as rotation and settlement. The excessive sliding showed that the retaining wall had a safety factor below than calaculated in the initial stability analysis, indicating the need for a reanalysis to find the causal factor for the instability.

2. Material and method

This research involved a study of soil, gabion stone fill and measurements of the gabion retaining wall to provide data for a stability reanalysis and to define the geometry and deformation at the present condition of the wall

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2.1 Material

The material analysed included the backfill soil, foundation soil, and gabion stone fill, along with the type and specification of the gabion baskets used.

2.1.1 Soil and gabion stonefill

Physical and mechanical properties of soil were obtained from laboratory testing. Disturbed and undisturbed soil samples were collected collected from the site of the failed wall.

Physical and mechanical properties of soil and gabion stone fill are shown in Table 1 below**:**

Table1. Properties of soil and gabion stone fill

Backfill soil and foundation soil	
physical	mechanical
Specific Gravity(Gs)	Unit weight
Natural water content(wn)	Internal friction angle
Atterberg limit	Cohesion
Grain size distribution	Undrained shear strength (Cu)
Gabion stone fill	
Unit weight	

2.1.2 Gabion basket

The gabion basket type used was fabricated from wire mesh with the properties from available manufacturer data (PT Jongka), as shown in Table 2 below:

Table2. Jongka Gabion basket properties

Dimension (cm)	Wire diameter (mm)	Coating: Alumunium Zinc (gr/ m^2	Wire strength
200x100 x50	side	290	41- 53 kgf/mm^2
	mesh	275	
	lacing	240	
Mesh aperture	80x100 mm		

2.2 Method

2.2.1 Soil properties determination

Determination of soil properties was conducted in the laboratory and testing method for each property show in Table 3.

Table 3 Properties determination method

Properties	Testing method	Sample type
Specific gravity(Gs)	SNI 1964:2008	disturbed
water Content w_n)	SNI 1965:2008	undisturbed
Grain size distribution	SNI 3423:2008	disturbed
Atterberg limit	SNI1966:2008	disturbed
Unit weight (γ)	AASHTO T233-1	undisturbed
Internal friction angle (Ø') Cohesion ')	SNI 2813:2008 (direct shear test)	undisturbed
Unconfined compressive strength	SNI 3638:2012	Disturbed

2.2.2 Gabion stone fill properties

In order to calculate the self weight of gabion retaining wall, a unit weight determination test was conducted by weighing a large box (0,45m x 0,45m x 0,60 m) filled with 15/20cm – 20/30 cm angular andesite stone (approximately same size and density as gabion stone fill in field) and dividing by volume of box.

2.2.3 Field survey

A field survey determined the slope of the backfill surface, retaining wall geometry and deformation using triangulation from some benchmark points on the roadside and critical points on the top of gabion wall. It was found the top of the backfill on the backside of the wall had consolidated by an average of 30 cm, and this was useful to estimate initial unit weight of the back fill at initial condition. Measurement was also made of the the size and shape of the gabion wall cross-section.

2.2.4 Active lateral earth pressure of backfill soil

Active lateral earth pressure determinations were made based on Rankine's theory using the soil's mechanical properties as measured by the laboratory test and reviewed in initial and present condition.

2.2.4.1 Initial condition

Considering that sliding failure occurred within 12 hours of completion of construction and the backfill soil was in unconsolidated condition, active lateral earth pressure was analyzed in an unconsolidated and undrained condition [7]:

$$p_0 = \gamma_0 z - 2c_{u0} \qquad (1)$$

Where:
p_0 is active lateral earth pressure at initial condition, (t/m^2)
γ_0 is unit weight of backfill soil at initial condition(t/m^3)
$c_{u0}$is undrained shear strength of backfill soil at initial condition,(t/m^2)
z is depth (measured from the top of backfill) (m)

Initial unit weight (γ_0) was estimated by multiplying the present unit weight (γ) by ratio of the present to the initial height of backfill (H/H_0), as illustrated in Fig 1.

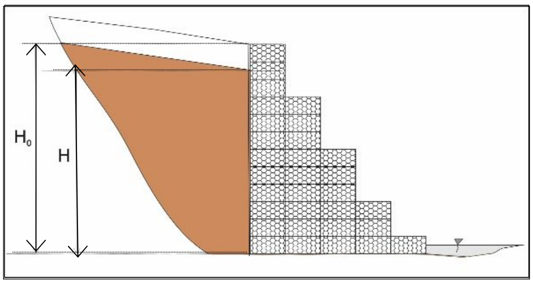

Fig. 1. back fill height at present (H) and initial (H_0) condition

$$\gamma_0 = \frac{H}{H_0} \tag{2}$$

In common practice, only active pressure distribution between z_c and z = H is considered for total lateral active calculation, because there is no contact between soil and the wall in the tensile zone[7], active pressure distribution is shown in Fig 2.

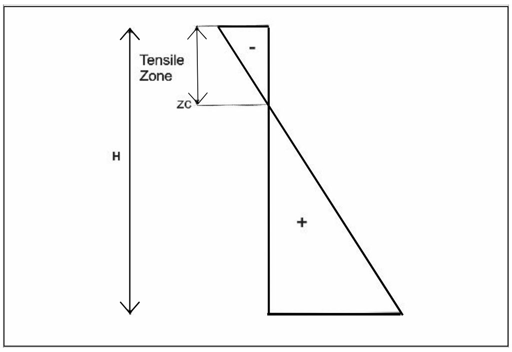

Fig. 2. active lateral pressure diagram

And total active force in undrained condition P_0 [7]

$$P_0 = \frac{1}{2}\gamma_0 - 2c_{u0}H_0 + 2\frac{c_{u0}^2}{\gamma_0} \tag{3}$$

2.2.4.2 Present condition

Active lateral earth pressure p_h in present condition determined using effective shear strength parameter (Ø') and (c') [7]

$$K_a = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi'}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi'}} \tag{4}$$

And active lateral force (Ph) is equal to [7]:

$$P_h = \frac{1}{2}\gamma K_a H^2 - 2\sqrt{K_a}\, c' H + 2\frac{c'^2}{\gamma} \tag{5}$$

Where:
K_a is coefficient of active lateral earth pressure
β is angle of backfill soil(°)
ϕ' is internal friction angle of backfil soil(°)
P_h is Total active lateral earth pressure, (t)
c' is cohesion of backfill soil, ((t/m^2)

2.2.5 Stability safety factor

Stability safety factor (SF) was examined for overturning, sliding, and bearing capacity and reviewed for initial and present condition.

2.2.5.1 Overturning stability SF

Initial condition[8]:

$$SF_{\text{overturning}} = \frac{\Sigma M_r}{\Sigma M_o} = \frac{\Sigma Wx + P_{0v} B}{P_{0v}(P_{0v} - z_c)/3} \tag{6}$$

Present condition[8]:

$$SF_{\text{overturning}} = \frac{\Sigma Wx + P_v B}{P_h(H' - z_c)/3} \tag{7}$$

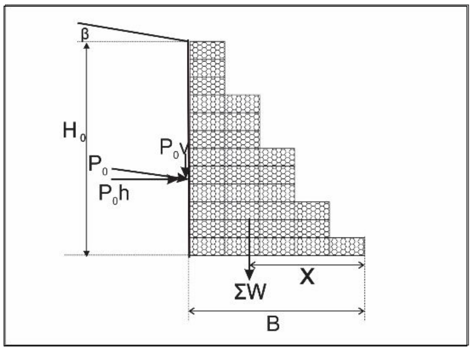

Fig.3a. illustration for overturning stability (initial)

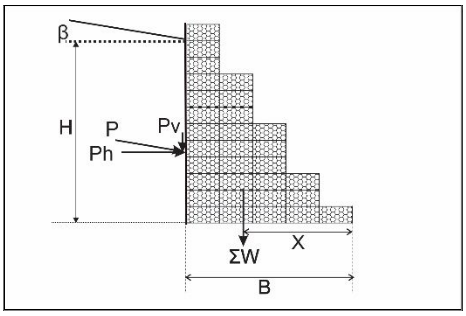

Fig.3b. illustration for overturning stability (present)

Where:
ΣMr is Sum of resisting moment (tm')
ΣM_0 is Sum of overturning moment (tm')
ΣW is Total weight of retaining wall(t/m')
X is distance from the central gravity of wall to the front edge of the base(m)

2.2.5.2 Sliding stability SF

$$SF_{\text{sliding}}[8] = \frac{\Sigma F_r}{\Sigma P_h} \tag{8}$$

Where:
ΣFr is Sum of sliding resistance force (t)
ΣPh is Sum of horizontal force. (t)
The assumed sliding resistance force was that from friction between the gabion wire mesh at the base and foundation soil only assuming there is no direct contact between stone fill at the base and the foundation soil. The sliding force was assumed to be equal to the horizontal active force.

$$\Sigma F_r\,[8] = \Sigma V\,\mathrm{tg}\delta \tag{9}$$

Where:
ΣV = total vertical force (t)

δ is the friction angle between soil and the wire mesh at the base, for which the friction coefficient between soil and smooth metal proposed by [9] was adopted.

2.2.5.3 Soil bearing capacity SF

Soil bearing capacity (q_u) determined according to the equations used by Hansen (1971) and Vesic (1975) [8] and pressure on soil foundation (q) determined according to Terzaghi, 2002)[8].
Where:

$$q_u = d_c i_c c N_c + d_q i_q D_f \gamma N_q + d_\gamma i_\gamma 0{,}5 B \gamma N_\gamma \tag{10}$$

$$q = \frac{V}{B}\left(1 \pm \frac{6e}{B}\right) \text{ if } e \le B/6 \tag{11}$$

$$q = \frac{2V}{3(B\text{-}2e)} \text{ if } e > B/6 \tag{12}$$

and :

$$SF_{\text{bearing}}[8] = \frac{q_u}{q} \tag{13}$$

Where:
q_u is ultimate bearing capacity of foundation soil (t/m^2)
q is pressure on foundation soil (t/m^2)
d_c, d_q, d_γ is depth factor
i_c, i_q, i_γ is load inclination factor
N_c, N_q, N_γ is bearing capacity factor
D_f is depth of foundation(m)

Soil bearing capacit safety factor reviewed for initial and present condition. Foundation soil pressure is calculated based on active lateral force for each condition and foundation soil mechanical properties.

2.2.6. Flow chart

Research methodology is shown on flow chart below:

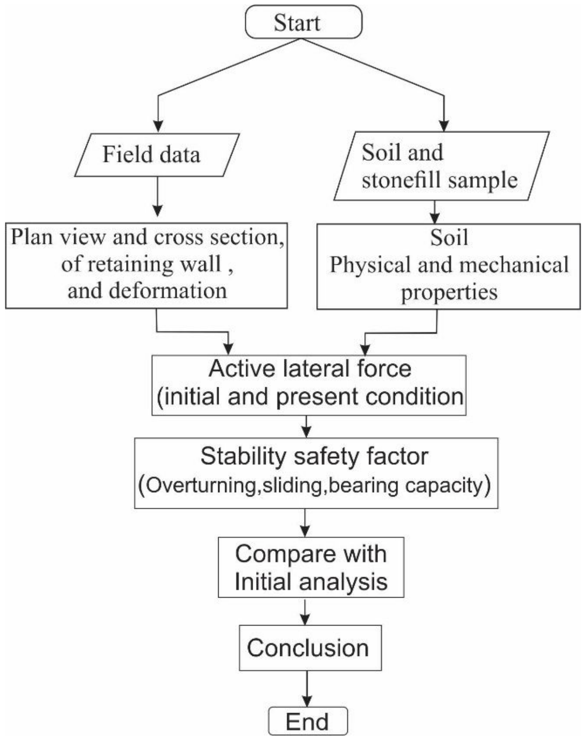

Fig.4 Research methodology flow chart

3. Results and Disscussion

3.1. Geometry and deformation

3.1.1 Geometry

The plan view geometry of gabion retaining wall before and after sliding is shown Fig 5a dan 5b below

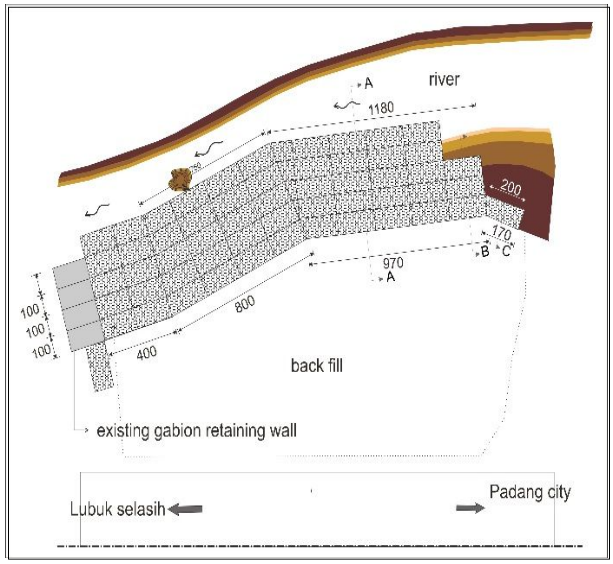

Fig.5a. Plan view of the gabion retaining wall before sliding

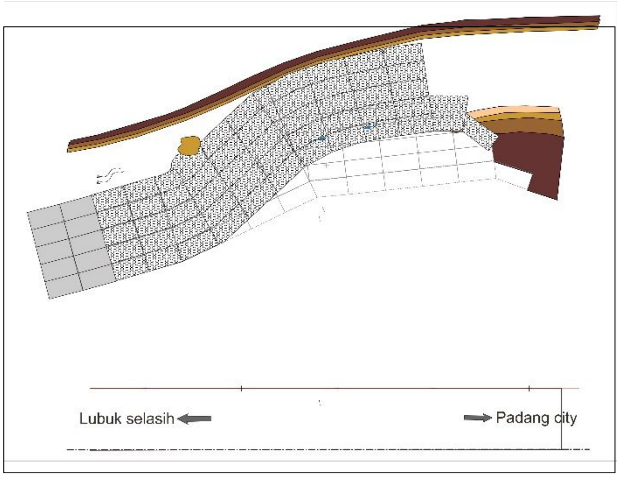

Fig.5b. Plan view of the gabion retaining wall after sliding

Fig 6a and 6b show the cross section of the gabion retaining wall before and after sliding

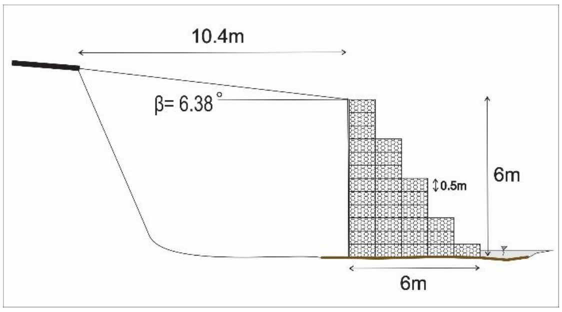

Fig.6a.cross section (before sliding)

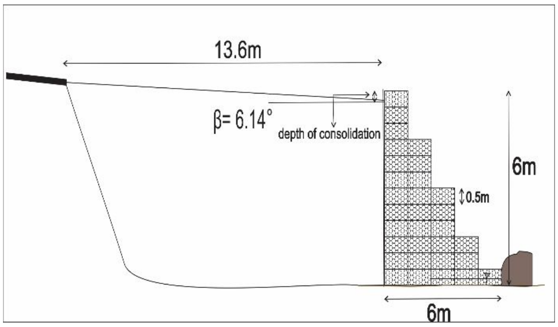

Fig. 6b. cross section (after sliding)

Fig 7a and 7b are photographs of gabion retaining wall during construction and after sliding.

Fig.7a. The gabion retaining wall during construction

Fig.7b. The gabion retaining wall after sliding

3.1.2 Deformation

Based on processed measurement data obtained from the field survey, the deformation is shown Fig 8 and Table 4 below:

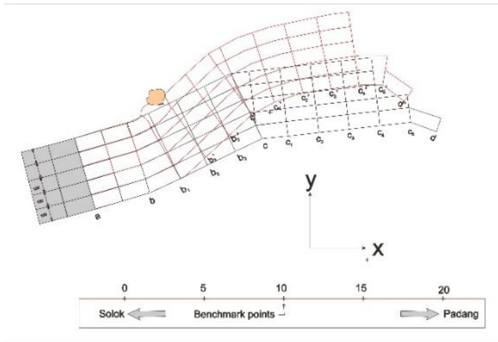

Fig .8.deformation measurement

Table 4..Deformation for each point

initial point	Present point	Deformation(m)	
		x	y
a	a'	0	0
b	b'	0	0
b 1	b 1’	0	0
b 2	b 2’	-0.105	0.497
b 3	b 3’	-0.39	0.99
c	c'	-0.64	1.52
c 1	c 1’	-0.86	2.13
c2	c2’	-0.98	2.47
c3	c3’	-1.15	2.54
c4	c4’	-1.311	2.55
c5	c5’	-2.24	2.36
c6	c6’	-1.62	2.95

3.2 Soil properties and stone fill unit weight

3.2.1 Soil physical properties (based on laboratory test result)

Physical properties for each soil sample was found to be as shown in Table 5 below:

Table 5..Physical properties of soil

properties		unit	result	
			Backfill	foundation
Spec.gravity (Gs)		-	2.629	2.539
Natural water content(w_n)		%	53.7	30.88
Grain size distribution	No.of sieve	-	% comm. passing	
	3”	%	-	100
	2”	%	-	56.2
	3/8”	%	-	30.6
	4	%	-	28.3
	10	%	100	23.9
	16	%	98.9	22.9
	20	%	98.7	22.6
	30	%	97	21
	40	%	95.4	19.9
	50	%	93.4	14.3
	80	%	90.8	13.6
	100	%	88.5	12.5
	200	%	84.2	9.3
Atterberg limit and PI	LL	%	59.82	42.18
	PL	%	37.2	31.06
	PI	%	22.62	11.12

Soil classification was determined from grain size distribution and atterberg limit data with reference to USCS. Backfill soil classify as MH soil and foundation soil as GP-GM soil.

3.2.2 Gabion stonefill

Gabion stonefill consist of angular andesite stone with average size of 15/25 to 20/ 35 cm. Unit weight determination conducted by large box filled with andesite stone with approximately same stone size and density with gabion in the field and was found to be 1.722 t/m^3.

3.2.3 Soil mechanical properties

Table 6a and 6b, show soil mechanical properties based on laboratory test results also mechanical properties for backfill soil determined for initial and present conditions.

Table 6a. Mechanical properties of the backfill soil

properties	Unit	Test result	
		initial	present
Unit weight	t/m^3	1.593	1.677
Internal friction angle (Ø’)	°	0	23.025
Cohesion(c’)	t/m^2	-	1.2
Unconfined compression strength (q_u)	t/m^2	3.24	6.56
Undrained shear strength (cu)	t/m^2	1.62	3.28
Consistency[8]	-	soft	medium

Table 6b. Mechanical properties of the foundation soil

properties	Unit	Test result
Unit weight	t/m^3	1.81
Internal friction angle (Ø’)	°	48.37
Cohesion(c’)	t/m^2	2.6
Unconfined compression strength (q_u)	t/m^2	27.53
Undrained shear strength (cu)	t/m^2	13.77
Consistency[8]	-	Very stiff

3.3 Stability calculation

3.3.1 Active lateral force

Active lateral force was determined for initial condition and present condition,and used equation (3) and (5) for calculation. Calculation results are shown in Table 7 below:

Table 7. Active lateral force

condition	unit	Active lateral force	
Initial (Po) (Eq.3)	t/m’	12,529	Po_h= 12,451
			Po_v= 1,392
Present (Ph) (Eq.5)	t/m’	4.647	Ph_h= 4.62
			Ph_v= 0.494

3.3.2 Stability safety factor

Stability safety factor was determined for initial and present condition for overturning, sliding and soil bearing capacity.

3.3.2.1 Overturning stability safety factor

Safety factor calculation follow eq(6) and (7) for each condition show in table 8 below:

Table 8. Overturning safety factor

condition	ΣMr (Tm/m’)	ΣMo (Tm/m’)	SF	SF min
Initial (Eq.6)	97.474	16.46	5.922	1.5
Present (Eq.7)	93.3	5.452	17.11	

3.3.2.2 Sliding stability safety factor

In sliding stability analysis, the sliding resistance force was calculated by eq (9), and the friction coefficient between foundation soil and gabion wire mesh was determined by an adopted friction coefficient (δ/Ø’) of 0.4 between granular cohesive soil and smooth metal as proposed by [9]. This meant the friction angle between wire mesh and foundation soil was 0.4x48.37=19,348º.

The Sliding stability safety factor for each condition is shown in Table 9 below:

Table 9. Sliding safety factor

condition	ΣFr (T/m')	ΣPh (T/m')	SF	SF min
Initial Eq.(8)	9.861	12.451	0.792	2
Present Eq.(8)	9.55	4.62	2.067	

3.3.2.3 Soil bearing capacity safety factor

Soil bearing capacity safety factor calculated by eq (10), (11),(12),and (13) for each condition and was as shown in Table. 10 below:

Table 10. Soil bearing capacity safety factor

condition	qu (T/m^2)	q (T/m^2)	SF	SF min
Initial	406.23	6.237	65.132	3
Present	1216.05	5.674	214.32	

3.4 Discussion

3.4.1 Comparison of soil and stone fill mechanical properties

In table 11 below, we can see the comparison of the soil and stone fill mechanical properties between initial analysis (before construction) and reanalysis. In initial analysis, soil mechanical properties was refered to [10-12]

Table 11. Soil mechanical properties comparison between initial analysis and reanalysis

Properties		Initial analysis (based on literature source)	Reanalysis (Based on laboratory test result)	
			Initial condition	Present condition
Unit weight (T/m^3)	bf	1.7	1.593 (estimated)	1.677
	f	1.7	-	1.81
	sf	1.7	1.722	1.722
Internal friction angle (°)	bf	25	-	23.025
	f	45	-	48.37
Cohesion (T/m^2)	bf	1	-	1.2
	f	0	2.6	2.6
Undrained shear strength (T/m^2)	bf	-	1.62	3.28
	f	-	13.77	13.77
Note: Bf: backfill soil, f: foundation soil, sf: stone fill				

From table. 11 above, we can see soil and stone fill mechanical properties in the initial analysis that was based on literature sources, were generally almost identical to the mechanical properties from the reanalysis based on laboratory test results excepting that in the initial analysis, undrained shear strength and soil mechanical properties for unconsolidated undrained condition (initial condition) was not determined.

3.4.2 Stability safety factor comparison

Comparison of the stability safety factors between initial analysis and reanalysis is shown in Table 12 below.

Table 12. Stability safety factor comparison between initial analysis and reanalysis

Stability	Safety factor		
	Initial analysis (pre construction)	reanalysis	
		Initial condition(2010)	Present condition (2019)
Over turning	13.335	5.922	17.11
sliding	3.15	0.792	2.067
Soil bearing capacity	58.74	65.132	214.32

In the initial analysis, active lateral force was determined based on effective shear strength of backfill soil alone and not calculated for the unconsolidated and undrained condition. This is considered a mistake because the clay soil was still in the undrained condition when the backfilling work was completed and a long time was needed to dissipate pore water pressure until inter grain contact with the soil was achieved, and effective shear strength of soil developed. So it was more appropriate to use undrained shear strength parameter in calculation.

In Table 12, the stability safety factor for each analysis is shown. For overturning stability, the safety factor has its greatest value in the present condition. This is because in the present condition the backfill has the lowest height and so cohesion of backfill soil has a bigger value than in the initial analysis making the lateral active force smaller.

In the initial condition, the overturning safety factor has smallest value as the active lateral force has the biggest value due to active pressure coefficient being equal to 1.

For sliding stability, reanalysis of the initial condition had the smallest safety factor and was lower than 1 because the friction coefficient in the initial condition had a smaller value than in initial analysis. Also, for the initial condition, the active pressure force had a larger value than in the other analysis.

The formula for the conventional sliding resistance force at the base of the retaining wall was taken from [8], where $\Sigma Fr = \Sigma Vtg\delta$ and δ= 1/3 Ø' – 2/3 Ø', and Ø'= internal friction angle of foundation soil. And if we use δ= 2/3 Ø'(as customary for masonry or concrete retaining walls) δ=32.247 º and $\Sigma Fr =$ 17.717 t/m' and SF sliding = 1,422 which is still less than the minimum requirement for the sliding safety factor . But we must consider, there is a difference between the interaction between soil and the base of the gabion retaining wall and between soil and the base of a stone masonry or concrete retaining wall.

Based on laboratory test results, it was found the internal friction angle of foundation soil had a larger value than in the initial analysis. In the present condition, the active lateral force is smaller, and this is the reason why the soil bearing capacity safety factor for the present condition is larger than in the other analyses.

All this demonstrates that the initial condition has lowest safety factor for all stability analyses and as it is lower than 1 or just 25.14 % of the initial analysis, this was why the gabion retaining wall was in an unstable state.

3.4.3 Deformation

Fig 7a and 7b show the difference in shape between the gabion wall before and after sliding and the resulting curvature. From table 4, we can see deformation of the gabion retaining wall varies from 0.49 m to 2.95 m from the Y- axis, and from -0.105 m to -1.65 m from the X-axis. Field observation showed there was no broken wire in the gabion baskets due to this large deformation or sign of tilting of the gabion baskets. This along with the large deformation is further evidence that the gabion retaining wall was unstable in sliding stability.

4. Conclusion

Based on stability reanalysis for present and initial conditions and deformation measurement the following conclusions can be drawn:

1. Stability analysis in initial condition resulted in the lowest safety factor for all stabilities due to the largest active lateral force resulting from the unconsolidated and undrained condition of backfill soil. This emphasises the necessity of performing the stability analysis in the unconsolidated and undrained condition as the gabion retaining wall experienced excessive sliding deformation while the soil was still unconsolidated and undrained just 12 hours after the construction was completed.
2. The sliding stability safety factor result from initial condition analysis is 0.792 or only 25.14% of the initial analysis indicating the gabion retaining wall was in an unstable state and liable to move.
3. Sliding resistance force was overestimated in the initial analysis as the friction coefficient mistakenly assumed good contact had been developed over the whole area of the base as it would have been for concrete or masonry retaining wall.
4. In the sliding stability analysis, it is more appropriate to use the friction coefficient between the gabion wire mesh and foundation soil only and assume no direct contact between gabion stone fill and the foundation soil especially when the foundation soil has a high bearing capacity.
5. If this considerations are applied in planning for future gabion stone fill projects the failure experienced in this instance could be avoided in future.

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